Bean Tunnel

Ground Movement Analysis & Engineering Impact Assessment

Balfour Beatty

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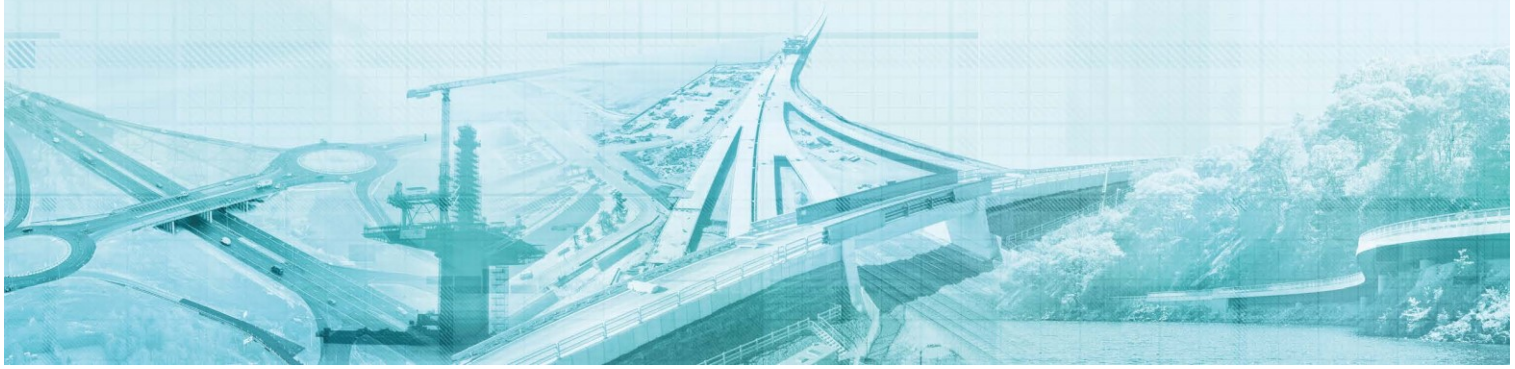

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1. Introduction and Purpose

Bean Tunnel is a proposed development to provide a Fastrack busway and a shared footway/cycleway route linking Eastern Quarry (proposed location for the Garden City) and Bluewater. This development will comprise of a new bore tunnel (Bean Tunnel) and associated road works to include bi-directional bus, cycling and pedestrian access between Eastern Quarry and Bluewater. The new tunnel will run under the existing B255 route.

Two haulage tunnels were constructed through the chalk spine supporting B255 Bean Road to facilitate earthworks movements during the construction of Bluewater Shopping Centre during the mid-1990's. These are located to the south of the proposed new Bean Tunnel which are currently not in use but are inhabited by bats. There are proposals to retain one of the tunnels for bat conservation and to infill the other tunnel, to reduce maintenance and potential risk of exposure to unauthorised access.

Tony Gee & Partners and their sub-consultant, Dr Sauer & Partners, as part of their design scope are to undertake a ground movement analysis and an assessment of the ground movement induced impact on the utilities which run along Bean Road.

The following has been analysed:

- The expected surface settlement at Bean Road due to tunnel construction. Settlements were calculated for tunnel top heading construction only, as well as for full tunnel construction.
- The expected impact on the Thames Water assets due to ground movement. That is, pipe strains, pipe joint rotations and pipe joint pull out.

Assets managed by UK Power Networks and SGN were not directly assessed but the presented results for ground movement and axial strain are equally applicable for these assets.

Revision P04 of this report introduces numerical analysis in addition to the previously presented empirical volume loss analysis. This was done for the following reasons:

- To consider the specific site characteristics (i.e. non-circular tunnel geometry and ground parameters)
- To consider elements (e.g. temporary sprayed concrete invert) of the chosen construction methodology (i.e. sprayed concrete primary tunnel lining), and
- To narrow down the wide range of ground movements which were calculated from the empirical analysis.

Revision P05 considers Thames Water comments which were received on 27th April 2021, and which were subsequently discussed in a meeting on 6th May 2021 [5].

Revision P06 of this report considers Balfour Beatty comments.

2. Ground Movement Analysis

The approximately 70m long Bean Tunnel will be approximately 11.7m wide and 9m high (outer tunnel dimensions, taken from the reference design developed by Jacobs [12]) and will be constructed through a spine of the Upper Chalk Formation. The spine is approximately 30m high and the exposed rock mass faces are near vertical on both sides. The vertical clearance between tunnel and the above B225 route is approximately 11.5m (Figure 1).

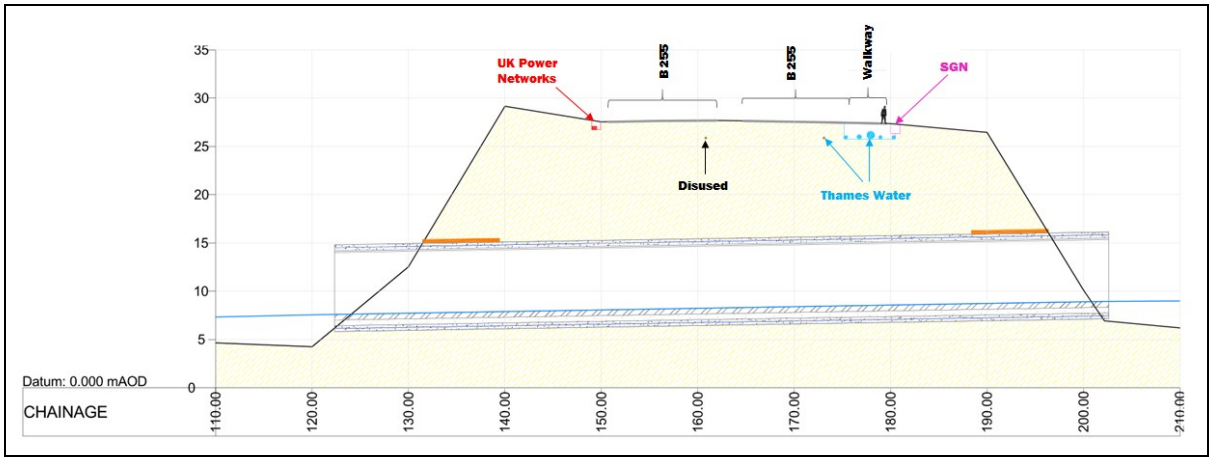


Figure 1 Section along the tunnel showing the geometry of the chalk spine, the B225 route on top of the spine, as well as the existing utilities

2.1. Empirical Volume Loss Analysis

The performed empirical ground movement analysis followed the methodology which bases on V_{loss} and K factors as presented in references [18], [19] and [21].

Published data of case studies where tunnels were constructed through chalk in the UK (references [18], [19] and [20]) was reviewed for the assessment of V_{loss} and K factors applicable for the project. Based on the review, the lower and higher limits of V_{loss} and K factor combinations considered are summarised in Table 1.

Table 1 V_{loss} and K factors for the empirical volume loss analyses.

Analysis	Volume loss (%)	K (-)
Lower limit	0.04	0.32
Higher limit	0.44	0.12

The ground surface settlement trough due to tunnel construction is well described by a Gaussian distribution curve (references [17], [18], [19] and [22]) with the following equation:

$$S_v = S_{max} \cdot exp(-y^2/2i^2)$$

With:

S_v	vertical settlement
S_{max}	maximum settlement at the tunnel centreline
y	horizontal distance from the tunnel centreline
i	horizontal distance between the tunnel centreline and the point of inflection of the settlement trough.

The parameter i can be calculated as (reference [21]):

$$i = K \cdot (Z_0 - Z_p)$$

With:

K	empirical constant
Z_0	depth of the tunnel axis
Z_p	depth of the utilities

The horizontal movements (S_{hf}) for the fully developed settlement trough and at the level of utilities for the fully developed settlement trough can be calculated as:

$$S_{hf} = \frac{y}{Z_0 - Z_p} \cdot S_v$$

Balfour Beatty's preference is to first construct the tunnel top heading over the full length, followed by the tunnel invert construction over the full length. This preference has been considered in the ground movement analysis and results are presented for top heading construction only, as well as for construction of the full tunnel profile.

2.1.1. Full tunnel constructed

The ground movement analysis results after construction completion of the full tunnel are summarised in Table 2, in which:

- Cover is the vertical distance between tunnel crown and ground surface, assumed to be 11.5m
- The empirical equations refer to the diameter of round tunnel shapes. The Bean Tunnel is characterised by an ovoid shape as per Jacobs reference design drawings with an excavation area in tunnel cross section of $A_{excavation}$=88.5m^2. An area equivalent tunnel diameter was considered for the ground movement analysis which calculates as follows:

$$D = \sqrt{\frac{4}{\pi} \cdot A} = 10.6\, m$$

- S_{max} is the maximum settlement expected at the level of utilities
- Z_0 is the depth of the tunnel axis below ground surface, which calculates as follows:

$$Z_0 = cover + radius\ of\ tunnel = 11.5 + \frac{10.6}{2} = 16.8m$$

- Z_p is the depth of the utilities, assumed to be 1.5m below ground surface.

Table 2 Parameter i and maximum settlement (S_{max}) after construction completion of the full tunnel.

Analysis	i (m)	S_{max} (mm)
Lower limit	4.9	2.9
Higher limit	1.8	84.6

2.1.2. Top heading constructed only

It is Balfour Beatty's preference to construct the full tunnel top heading before the start of tunnel invert construction. That means a smaller size tunnel will be completed and will be enlarged in a second phase. Ground movement will be monitored above the tunnel during construction which will allow for early verification of volume loss and trough width factors assumed with the ground movement analysis.

As described above, an area equivalent tunnel diameter was calculated from the top heading excavation area (i.e. 8.2m) provided with the reference design (i.e. $A_{excavation}$=52.5m^2).

The ground movement analysis results after top heading construction completion are summarised in Table 3.

Table 3 Parameter i and maximum settlement (S_{max}) after top heading construction completion.

Analysis	i (m)	S_{max} (mm)
Lower limit	4.5	1.9
Higher limit	1.7	54.5

2.2. Numerical Analysis

Due to the wide range in predicted maximum settlement (i.e. from 1.9mm to 84.6mm) in the empirical volume loss analysis, numerical analysis has been carried out by the use of a 2D finite element analysis (FEA) programme, Phase2 Version 8, from Rocscience Inc. It is to be noted that the numerical models are used to complement the empirical analysis based on case history data described in section 2.1.

The ground parameters used in the analysis are shown in Table 4 and are based on supporting data outlined in the footnote. The ground is modelled as elastic-plastic material using Mohr-Coulomb failure criterion. The sprayed concrete is modelled as elastic material with a Young's Modulus of 10MPa and a Poisson's Ratio of 0.2.

It is to be noted that boreholes are planned to be carried out from the top of the chalk spine. The gained information will be compared against the ground parameters shown in Table 4 for parameter verification.

The sprayed concrete lining is modelled as a series of beam elements and the number of beams depends on the applied mesh density. A lining thickness of 250mm has been considered in the analysis. Table 5 shows the individual modelling stages and when which part of the sprayed concrete lining is installed.

The geometry of the tunnel cross-section is modelled based on detailed design drawing [14], and the step-wise excavation and support of the tunnel has been simulated as shown in

Table 5.

Results in the form of contour plots are presented for the construction of the full tunnel profile as well as for top heading construction only in section 2.2.1 and 2.2.2. Additional contour plots, including those from parameter sensitivity analyses, are provided in Appendix A.

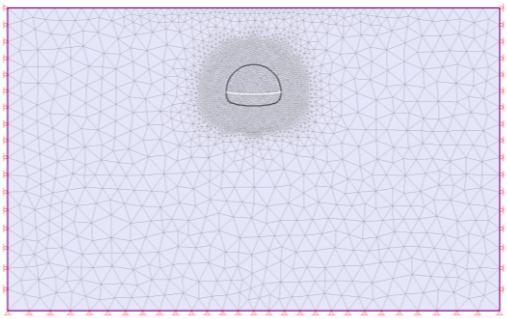
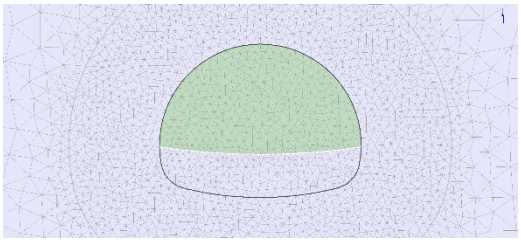
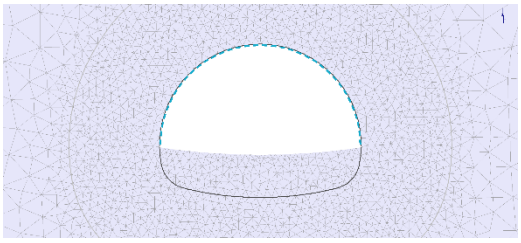
Table 4 Ground parameters used in the numerical analysis.

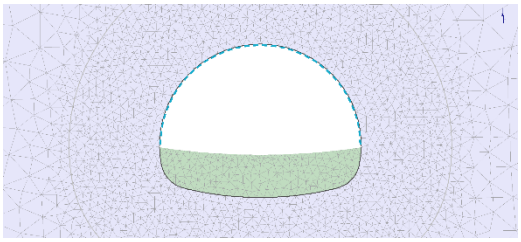
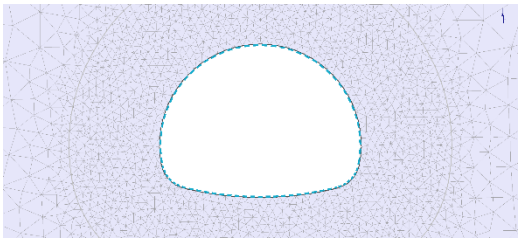
Ground Parameter	Chalk
Unit Weight (kN/m^3)	20
Horizontal to Vertical Stress Ratio, k_0	0.3[1]
Young's Modulus (MPa)	1000[2,5]
Poisson's Ratio	0.2

Ground Parameter	Chalk
Friction Angle (°)	34[3,5]
Cohesion (kPa)	50[4,5]

[1] k_0 value of 0.3 is low and conservative for the ground movement analysis. Case history data indicate back-analysed values between 0.5 and 0.6.
[2] Typical chalk Young's Modulus was considered. See also note 5.
[3] Typical range of friction angle for chalk was considered. See also note 5.
[4] The currrent slopes of the chalk spine were analysed and the cohesion of 50kPa is assessed as a suitable value for the chalk spine – see also Appendix B.
[5] Parameter sensitivity analyses were carried out and results indicate that cohesion have the most significant impact on ground movements – see Appendix C. The expected range of Young's Modulus and friction angle had insignificant impact.

Table 5 Modelling stages in the numerical analysis.

Stage	Description
1. Geostatic	
2. Excavation of Top Heading • 50% relaxation of ground stiffness	
3. Top Heading Primary Lining • 100% relaxation of ground stiffness • 250mm sprayed concrete	

Stage	Description
4. Excavation of Bench-Invert • 50% relaxation of ground stiffness	
5. Bench-Invert Primary Lining • 100% relaxation of ground stiffness • 250mm sprayed concrete	

2.2.1. Full tunnel constructed

Figure 2 shows the vertical ground movement plot from the FEA after construction completion of the full tunnel. The maximum settlement of 3mm is expected with a trough width of approximately 30m. The equivalent volume loss is approximately 0.05%.

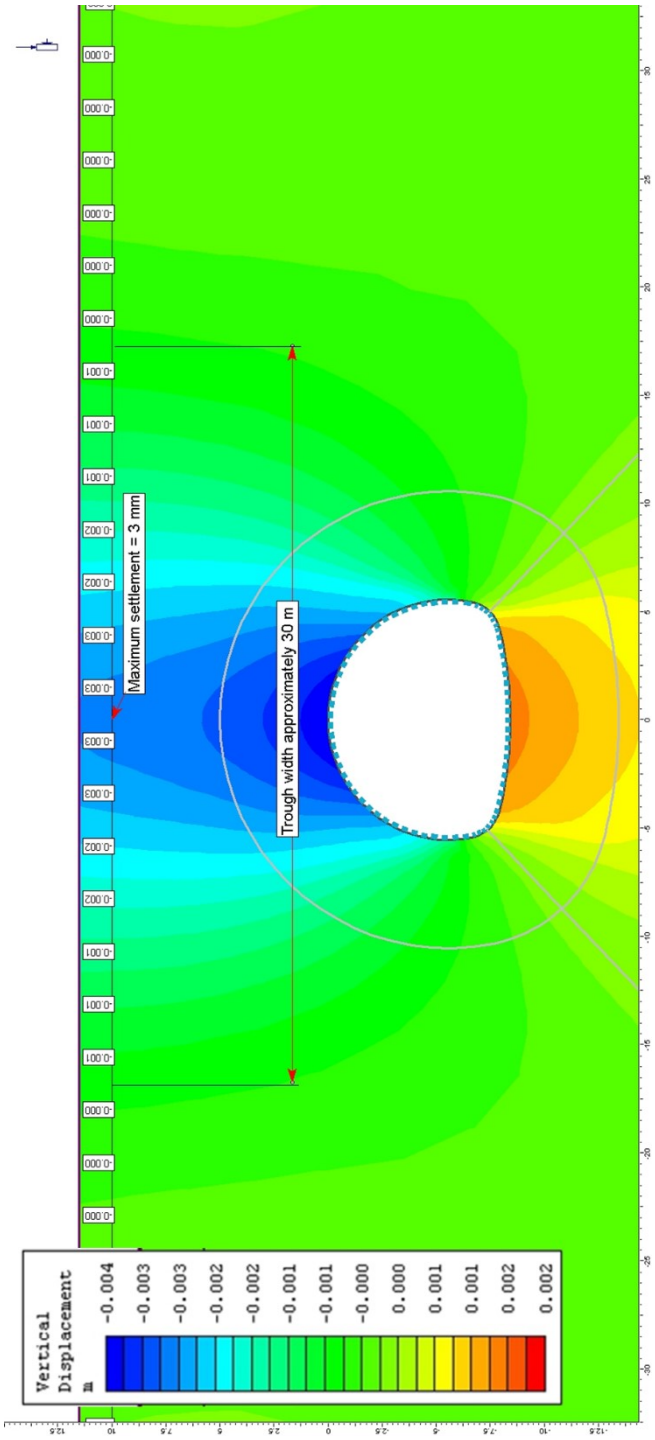


Figure 2 Vertical displacement plot after construction completion of the full tunnel.

2.2.2. Top heading constructed only

Figure 3 shows the vertical ground movement plot extracted from the FEA after top heading construction completion. The maximum settlement of 3mm is expected with a trough width of approximately 30m. The equivalent volume loss is approximately 0.05%.

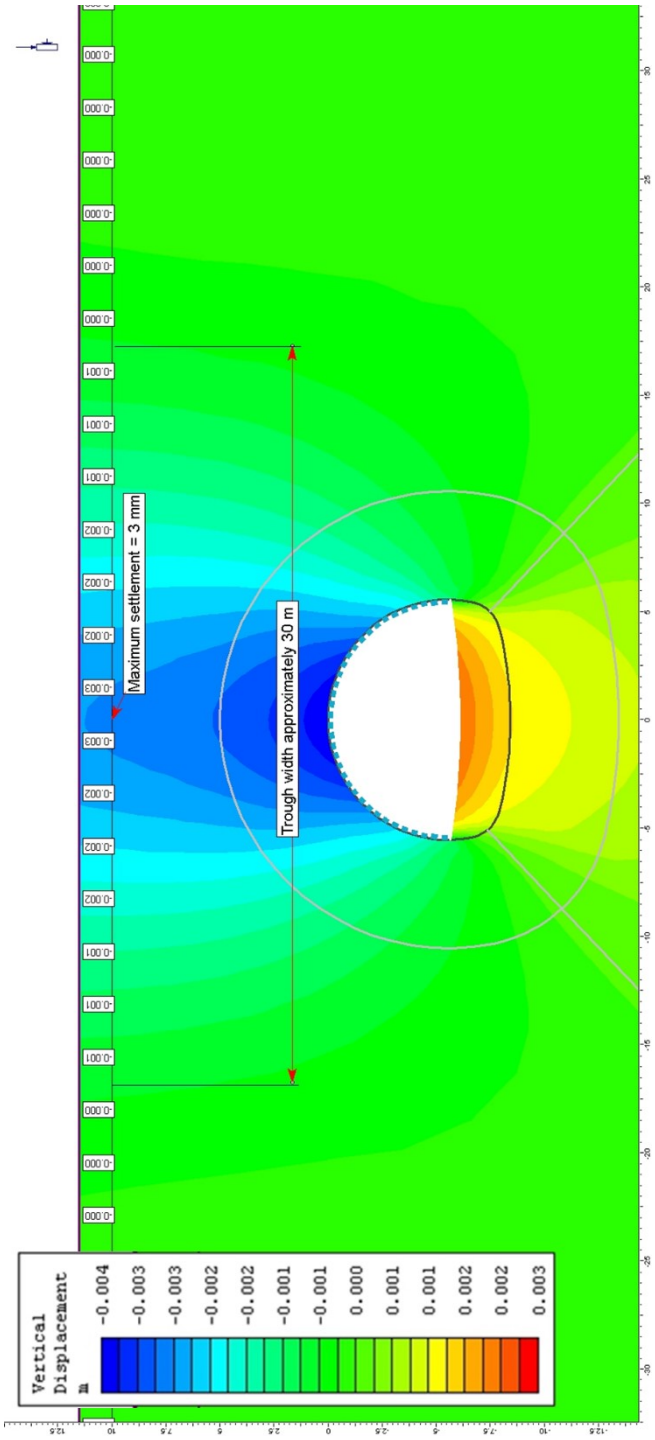


Figure 3 Vertical displacement plot after top heading construction completion.

3. Engineering Impact Assessment

The utilities located above the tunnel alignment and potentially impacted by the tunnel construction are the following (Figure 4), from west to east:

- UK Power Network assets
- Thames Water disused sewer
- Thames Water 225mm diameter vitrified clay sewer (material assumed based on Thames Water meeting, reference [2])
- Thames Water clean water 450mm diameter ductile iron pipe
- Thames Water clean water 315mm diameter Molecular Oriented PVC (PVC-O)
- Thames Water clean water 800mm diameter ductile iron pipe
- 2 No. Thames Water 355mm diameter black polyethylene pipe (MDPE), and
- SGN assets.

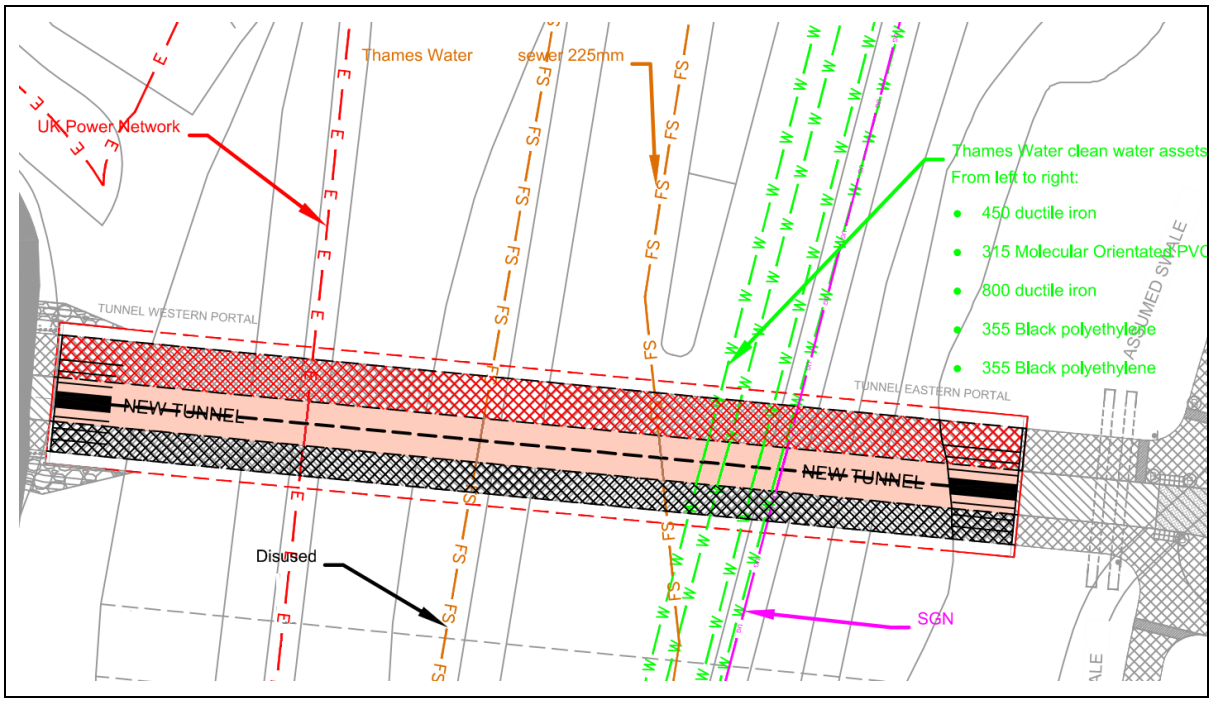


Figure 4 Tunnel alignment and existing assets in plan (references [7], [8], [9] and [11])

The assessment of the impact on existing assets was carried out for the Thames Water assets only and bases on the ground movement analysis presented in Section 2. Pipe strains, pipe joint rotations and pipe joint pull-out have been calculated in line with the requirements presented in Thames Water's guidance document (reference [6]). The calculated values have then been compared against the assessment criteria presented in the same document.

The exact depth of the existing assets below ground surface is unknown and was assumed to be 1.5m for the assessment. The condition of the Thames Water assets is expected to be good as the pipes are approximately 20 years old and are unlikely to have any burst history. This was shared by Thames Water in a meeting (reference [1]).

It was for the impact assessment conservatively assumed that the pipes follow the ground movements.

The exact location of the joints between pipes is not known and has consequently not been considered in the calculation of the joint rotations. It was assumed that pipe joints are located at the worst-case positions and that pipes are 3.66 m long (i.e. 12 ft) as proposed in reference [22].

For pipe incremental strains, it was assumed that joints are rigid and capable of transferring bending moments. Soil-pipe interaction factors were used to calculate axial pipe strains from the predicted ground strains.

In the following subsections of the report, results are presented in the format xx.xx (yy.yy – zz.zz), where xx.xx represents the predicted value based on numerical analysis; yy.yy and zz.zz represent lower and higher limits from the empirical analysis.

3.1. Pipe strains

Pipes are sensitive to increased tensile strain caused by tunnelling. In general, compressive strains are less critical. The total strain arises from two components: bending strain due to curvature, and axial strain.

It was assumed that the buried pipes deform following the greenfield ground movement profile (reference [18]) as calculated in Section 2. Axial and bending strains of the pipes were calculated based on the expected pipe deformation.

In Figure 5, ground movements and horizontal strains (equal to the axial strain up to a factor RF explained in 3.1.1) are plotted against the distance from the tunnel centreline for a pipe depth of 1.5m. In Figure 6, axial and bending strains (explained in 3.1.1 and 3.1.2) are plotted. In Figure 7, combined tensile and compressive strains are plotted. The maximum combined strains is then shown in Figure 8.

Positive strains indicate tension and negative strains indicate compression. Superposition of horizontal and bending strains confirm the worst-case tension strains to be 9.3m to either side of the tunnel centreline (i.e. $y=\sqrt{3}\cdot i$) and the worst-case compression strains to be at the centre of the settlement trough ($y=0$).

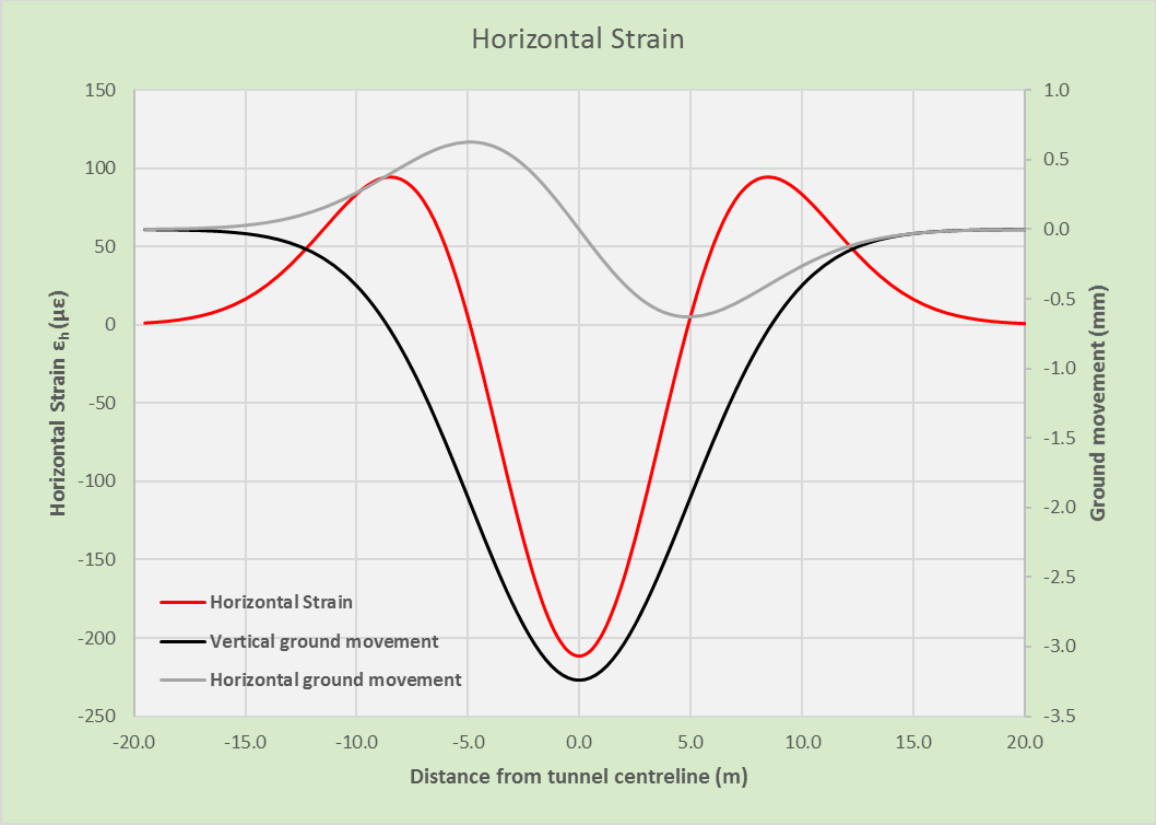
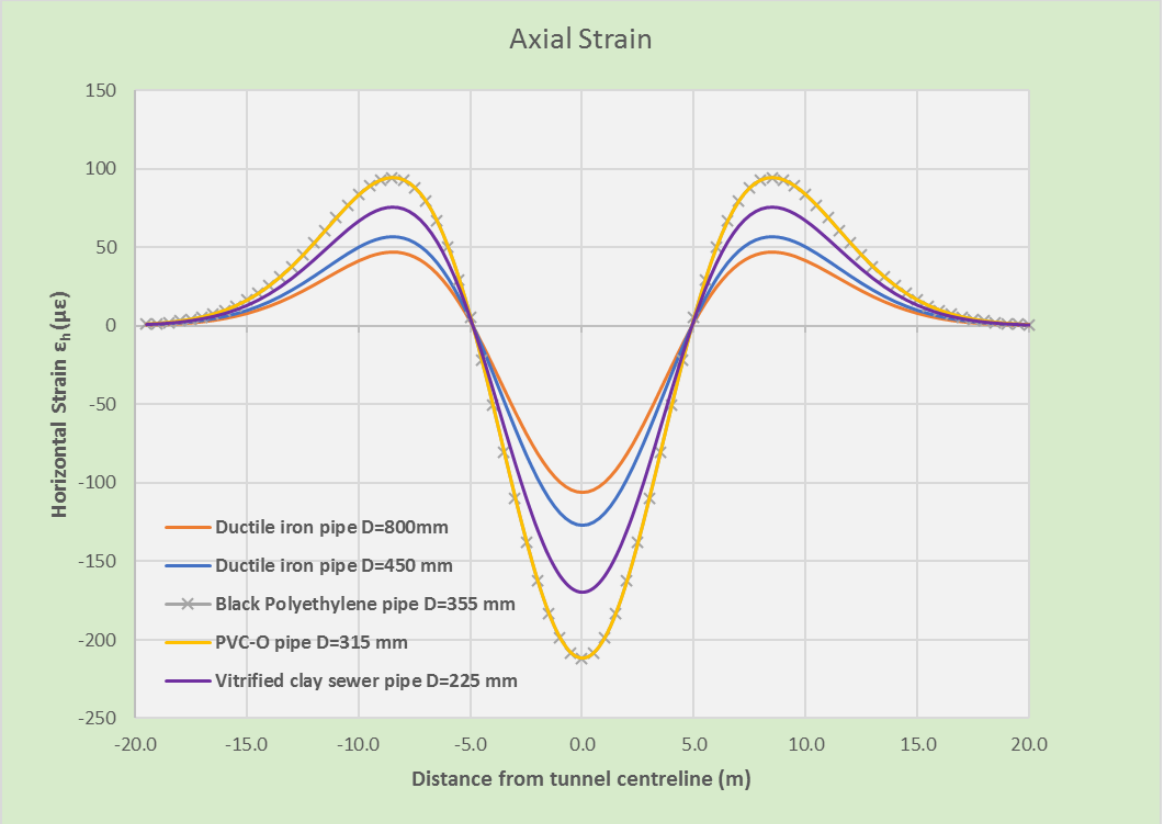


Figure 5 Vertical ground movement, horizontal ground movement and horizontal strain plotted against the distance from the tunnel centreline.



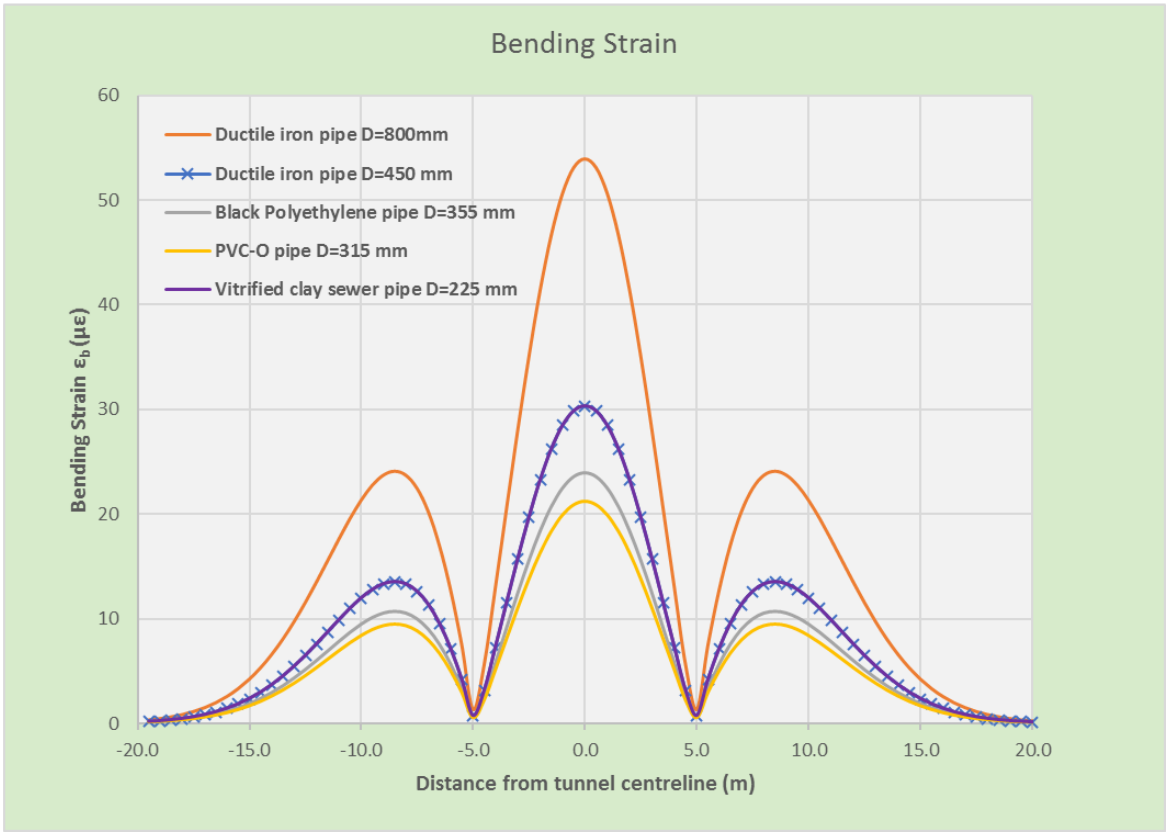


Figure 6 Axial and bending strains plotted against the distance from the tunnel centreline.

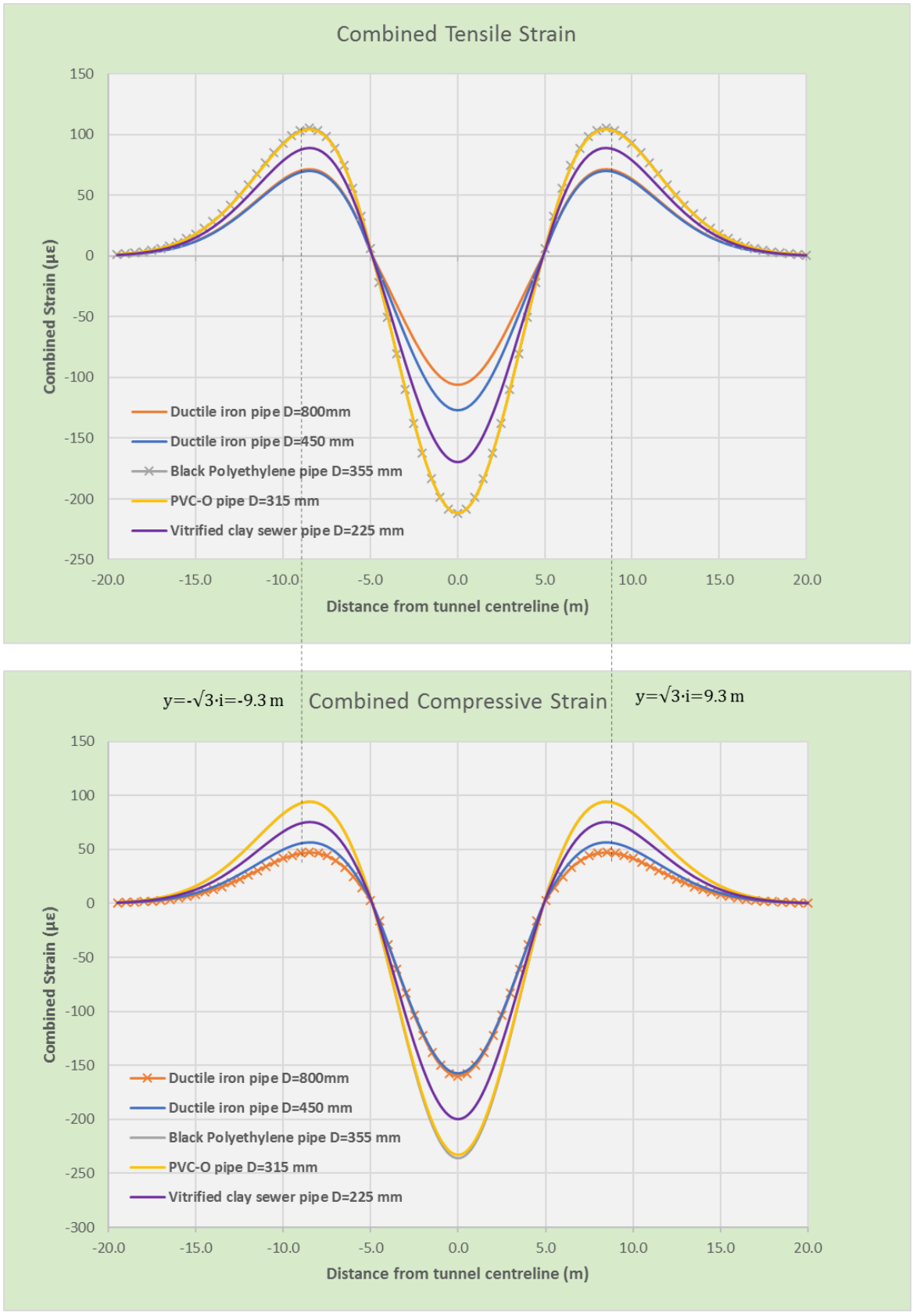

Combined Tensile Strain
Combined Strain (με)
Distance from tunnel centreline (m)
Ductile iron pipe D=800mm
Ductile iron pipe D=450 mm
Black Polyethylene pipe D=355 mm
PVC-O pipe D=315 mm
Vitrified clay sewer pipe D=225 mm
y=-√3·i=-9.3 m
Combined Compressive Strain
y=√3·i=9.3 m
Combined Strain (με)
Distance from tunnel centreline (m)
Ductile iron pipe D=800mm
Ductile iron pipe D=450 mm
Black Polyethylene pipe D=355 mm
PVC-O pipe D=315 mm
Vitrified clay sewer pipe D=225 mm

Figure 7 Combined tensile and compressive strains plotted against the distance from tunnel centreline.

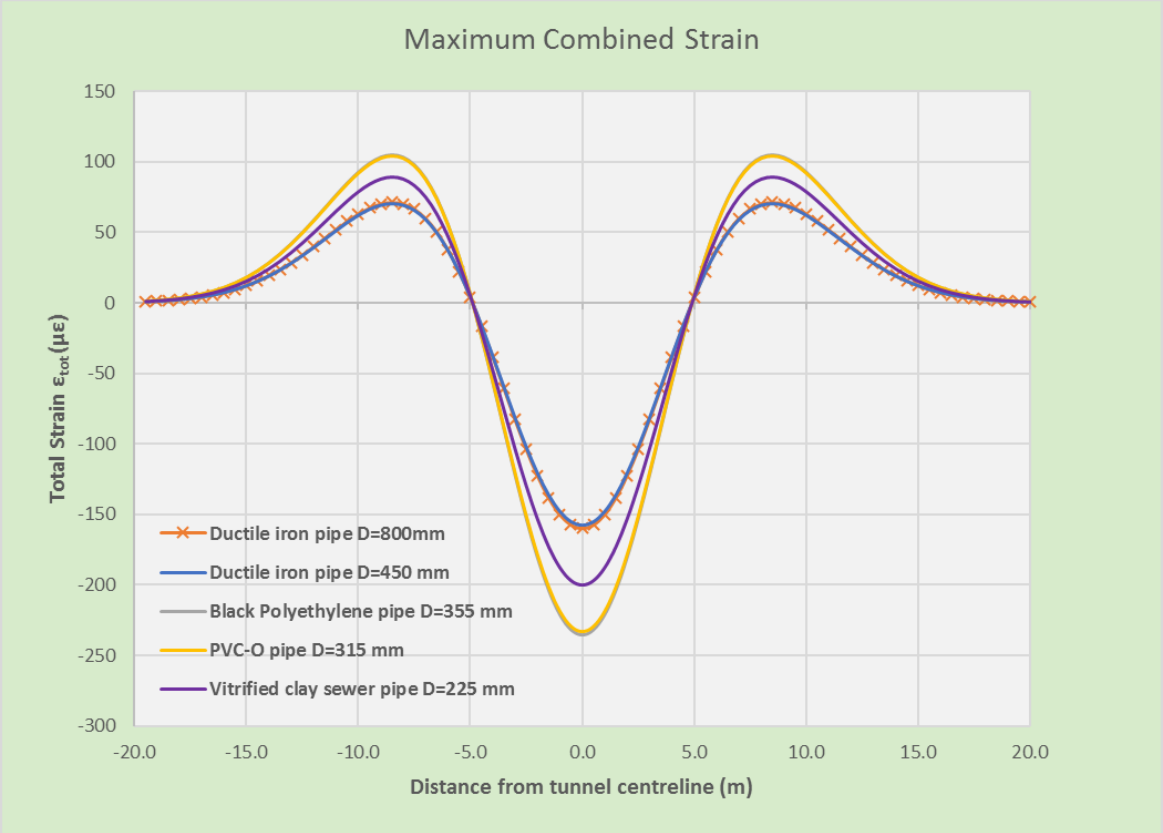


Figure 8 Maximum combined strain plotted against the distance from the tunnel centreline.

3.1.1. Axial Strain

The axial strains are calculated from the horizontal strains considering a reduction factor as presented in reference [17]. The reduction factor is a function of the pipe geometry, the Young's Moduli of the pipe and the soil surrounding the pipe, and it makes no allowance for slip between the pipe and the soil.

$$\varepsilon_{axial} = RF \cdot \varepsilon_h$$

With:

ε_{axial}	the axial strain due to ground movements
RF	the strain Reduction Factor (reference [17]), and
ε_h	the ground horizontal strain, that is the derivative with respect to y of the horizontal displacement S_h and given by the following equation:

$$\varepsilon_h = \frac{dS_h}{dy} = \frac{S_v}{Z_0 - Z_p} \cdot \left(1 - \frac{y^2}{i^2}\right)$$

Based on reference [17], the reduction factor RF was assumed to be 0.5 for the 800mm ductile iron pipe, 0.6 for the 450mm ductile iron pipe, 0.8 for the 225mm vitrified clay sewer pipe and 1.0 (i.e. no reduction applied) for all other pipes. The same axial strain reduction factor has been used for both tension and compression.

3.1.2. Bending strain

The bending strain (ε_b) is calculated from the classical beam equations and is a function of the outer pipe diameter (D) and the longitudinal radius of curvature (R).

Bending strain at pipe extrados:

a. ε_b = D/R for pipes materials concrete, masonry, and vitrified clay, and

b. ε_b = D/2R for other pipe materials (e.g. metal, MDPE and PVC-O)

The longitudinal curvature (R) in a generic point y=y_n is calculated using the following equation:

$$R_n = \delta^2 / \{2 \cdot S_{v,n} - (S_{v,n-1} + S_{v,n+1})\}$$

With:

δ	the interval between data points and set to δ = 0.1m for this assessment
$S_{v,n}$	the vertical displacement corresponding to y=yn
$S_{v,n-1}$	the vertical displacement corresponding to y=yn-δ
$S_{v,n+1}$	the vertical displacement corresponding to y=yn+δ.

3.1.3. Total strain

The total strain at any pipe location is the sum of axial strain, ε_{axial} and bending strain, ε_b.

$$\varepsilon_{tot} = \varepsilon_{axial} + \varepsilon_b$$

The maximum compressive strain occurs at the centre of the settlement trough (i.e. y=0) and the maximum tensile strain occurs at the inflection points of the settlement trough (i.e. y=√3·i).

Table 6 summarises the calculated maximum strain values for compression and tension and compare the values with the assessment criteria values presented in Thames Water's guidance document (reference [6]) as well as values taken from reference [20]. The guidance document provides assessment criteria values for the ductile iron (i.e. lead yarn joint assumed as worst case) and the vitrified clay sewer but not for the other materials. The corresponding values for the molecular oriented PVC and the black polyethylene pipes were taken from reference [20].

Table 6 Total calculated strains and their comparison against assessment criteria (limits from the empirical analysis shown in parenthesis)

Pipe	Max Increase in Strain			
	Tension (με)		Comp'n (με)	
	Predicted value	Assessment criteria	Predicted value	Assessment criteria
Ductile iron D=800 mm[1]	70 (14 - 1459)	500	160 (142 - 12780)	700
Ductile iron D=450 mm[1]	69 (13 - 1022)	500	158 (140 - 8950)	700
Black Polyethylene (MDPE) pipes[2]	104 (20 - 1138)	50000	236 (210 - 9970)	50000
PVC-O[2]	102 (20 - 1081)	25000	233 (207 - 9469)	25000
Vitrified clay sewer[1]	81 (17 - 1148)	80	200 (178 - 10055)	400

[1] Assessment criteria from Thames Water.
[2] Assessment criteria from reference [20].

3.2. Joint rotation

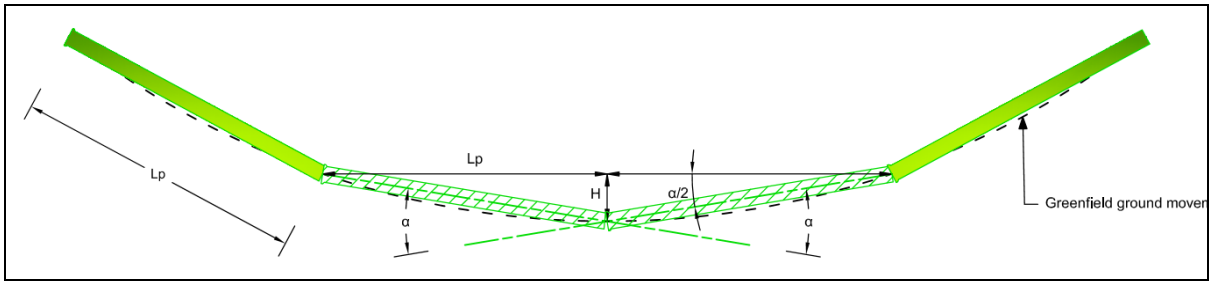


Figure 9 Joint rotation α

Based on reference [22], the worst-case pipe joint rotation considering a pipe length of Lp = 3.66m (i.e. 12 ft) can be calculated from the following equations (see also Figure 9):

$$\frac{H}{L_p} = \tan\frac{\alpha}{2}$$

With $H = S_{V,0}-S_{V,Lp}$ and therefore:

$$\alpha = 2 \cdot \tan^{-1}\left(\frac{S_{V,0} - S_{V,Lp}}{L_p}\right)$$

Table 7 shows the calculated joint rotation values and compares them with the assessment criteria values presented in Thames Water's guidance document (reference [6]) as well as values taken from reference [18]. The guidance document provides assessment criteria values for the ductile iron (i.e. lead yarn joint assumed as worst case) but not for the other materials. The corresponding values for the molecular oriented PVC and the black polyethylene pipes were taken from reference [18].

Table 7 Calculated joint rotation values and their comparison against assessment criteria (limits from the empirical analysis shown in parenthesis)

Pipe	Rotation (◦)	
	Predicted value	Assessment criteria
Ductile iron D=800 mm[1]	0.03 (0.02 - 2.30)	0.5
Ductile iron D=450 mm[1]	0.03 (0.02 - 2.30)	0.5
Black Polyethylene (MDPE) pipes[2]	0.03 (0.02 - 2.30)	2.5
PVC-O[2]	0.03 (0.02 - 2.30)	2.5
Vitrified clay sewer[1]	0.03 (0.02 - 2.30)	2.0

[1] Assessment criteria from Thames Water
[2] Assessment criteria from reference [18]

3.3. Joint pull-out

Pipe joint pull-out can be calculated considering the resultant axial strains of two consecutive segments (see also Figure 10):

$$PO = P_{length} \cdot \Sigma\varepsilon_{axial}$$

With:

PO	pull-out
P_{length}	length of pipe segment
$\Sigma\varepsilon_{axial}$	sum of axial strain along two adjacent pipe segments

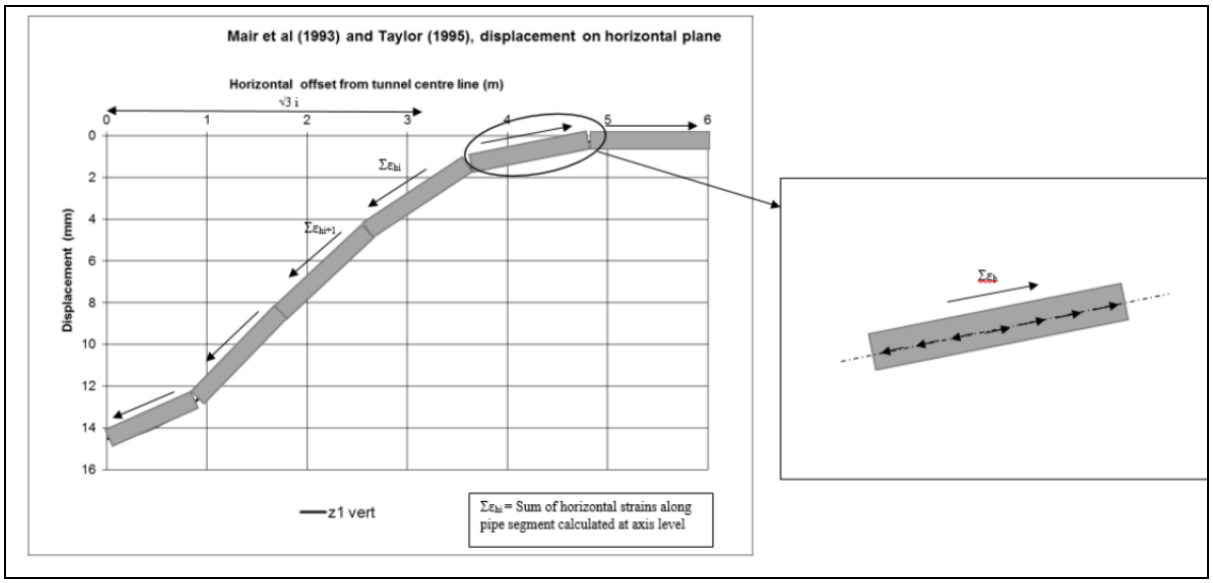

Mair et al (1993) and Taylor (1995), displacement on horizontal plane
Horizontal offset from tunnel centre line (m)
√3 i
Displacement (mm)
z1 vert
Σεhi= Sum of horizontal strains along pipe segment calculated at axis level

Figure 10 Strain resultant along a pipe extracted from reference [22]

Table 8 shows the calculated joint pull-out values and compares them with allowable pull-out values from Thames Water and reference [18]. Lead-yarn joints were assumed for ductile iron pipes and rubber gasket joints were assumed for the other pipe materials.

Table 8 Calculated joint pull-out values and their comparison against allowable pull-out values (limits from the empirical analysis shown in parenthesis)

Pipe	Pull Out (mm)	
	Predicted value	Assessment criteria
Ductile iron D=800 mm (lead-yarn joint assumed)[1]	0.28 (0.2 - 3.1)	5
Ductile iron D=450 mm (lead yarn joint assumed)[1]	0.34 (0.3 - 3.8)	5
Black Polyethylene (MDPE) pipes (rubber gasket joint assumed)[2]	0.56 (0.5 - 6.3)	25
PVC-O (rubber gasket joint assumed)[2]	0.56 (0.5 - 6.3)	25
Vitrified clay sewer (rubber gasket joint assumed)[2]	0.45 (0.4 - 5.0)	25

[1] Assessment criteria from Thames Water
[2] Assessment criteria from reference [18]

4. Existing Assets Monitoring

4.1. Trigger values

The existing assets monitoring trigger values proposed are as followed:

- Amber trigger is the predicted movement from the FEA (see section 2.2);
- Red trigger is a value chosen between amber and black trigger; and
- Black trigger is the maximum allowable movement back calculated using Thames Water acceptance criteria in section 3.

Back calculation of the maximum allowable movement is described below:

- Based on Table 6, Table 7 and Table 8, the ratio of acceptance criteria to predicted values were calculated.
- The 800mm ductile iron pipe has the lowest ratio of 4.38 (i.e. 700/160 from Table 6).
- Maximum allowable movement is then back calculated as 12mm, i.e. movement which will result in a ratio of acceptance criteria to predicted value of 1.0 for the 800mm ductile iron pipe. Detailed calculation is provided in Appendix C.

Summary of proposed trigger values are shown in Table 9.

Table 9 Existing assets monitoring trigger values.

	Trigger Value (mm)		
	Amber	Red	Black
Bean Road surface	4	8	12

The detailed design drawing showing instrumentation and monitoring arrangements for Bean Road is included in Appendix D of this report.

4.2. Contingency Measures

An Emergency Preparedness Plan (EPP) will be prepared by the Contractor which will define the actions to be taken in the case of monitoring trigger value breaches. Table 9 shows the proposed monitoring trigger values.

Contingency measures proposed in the detailed design include:

- Temporary sprayed concrete invert;
- Spiling;
- Pocket excavation; and
- Face wedge

5. Conclusion

5.1. Ground Movement Analysis

A ground movement analysis was carried out to assess the expected ground movements due to construction of the Bean Tunnel. The following points summarise key input parameters and analysis results:

- An **empirical volume loss analysis** was carried out considering the following range of volume loss factor and a trough width factor:
    - Volume loss factor between 0.04% and 0.44%, and
    - Trough width factor between 0.12 and 0.32.
- The above values have been assumed based on case studies of mined tunnels constructed in chalk and in the UK.
- The ground movement analysis was carried out for the full tunnel cross section and for the tunnel top heading construction only. This was to consider Balfour Beatty's preferred tunnel excavation and support sequence.
- The depth of utilities below ground surface was assumed to be 1.5 metres.
- The following ranges were analysed from the empirical volume loss analysis:
    - Maximum settlement at tunnel centreline in plan between 2.9mm and 84.6mm, and
    - Settlement trough width between 10.8m and 29.4m.
- Due to the wide range in predicted maximum settlement from the empirical volume loss analysis, **numerical analysis** has been carried out. The numerical analysis considers the latest available geotechnical parameters, detailed design tunnel cross-section, as well as the step-wise excavation and support construction sequence.
- The maximum settlement from the numerical analysis was analysed to be 3mm and is expected to occur at the location of the tunnel centreline in plan. The corresponding settlement trough is expected to be approximately 30m wide.

Monitoring of the ground surface will be required during construction to verify the assumptions the ground movement analysis is based on.

5.2. Engineering Impact Assessment

An engineering impact assessment was carried out in line with the requirements in Thames Water's document "Guidance for working near our assets" (reference [6]). The following points summarise key input parameters and analysis results:

- Pipe diameters, locations and materials were used as per the Thames Water searches and Jacobs reference drawing on utilities (references [7], [8], [9] and [11). The sewer was assumed to be of vitrified clay as per Thames Water's meeting notes (reference [2]).
- Pipe strains, pipe joint rotations and pipe joint pull-out were analysed and compared against the assessment criteria presented in Thames Water's guidance document. It is to be noted that this document covers ductile iron and vitrified clay but none of the other materials.
- Literature values were used as assessment criteria for molecular oriented PVC and black polyethylene pipes (references [18] and [20]).
- The ground movement results from the numerical analysis were used for the engineering impact assessment.
- The analysed **pipe strain** values passed the assessment for the ductile iron, PVC and polyethylene pipes. However, the following pipes did not pass the assessment (also refer to Table 4):
    - The vitrified clay sewer (D=225mm) pipe did not pass in tension.
- The analysed pipe strain value for the vitrified clay pipe exceeds the assessment criteria presented in Thames Water's guidance document and adequate risk mitigation measures will need to be put in place (e.g. installation of an inliner).
- All analysed **pipe joint rotations** and **pipe joint pull-outs** passed the assessment criteria.
- The **risk of significant damage** to the black polyethylene and molecular oriented PVC pipes can be considered very low.
- The risk of significant damage to the ductile iron pipes can be considered low. The following risk mitigation measures are recommended depending on the direction of tunnel construction:
    - Tunnel construction from the East: Installation of a temporary sprayed concrete invert until construction has passed the influence area of the pipes (i.e. approximately 35 m from the portal), and
    - Tunnel construction from the West: Verification of the ground movement analysis assumptions through ground surface monitoring and data interpretation. Installation of a temporary sprayed concrete invert in the influence area of the pipes should the observed movements be significantly greater than what was assumed in this assessment. The temporary invert is a contingency measure (also refer to section 4.2 of this report) in this case and the area of potential installation is the tunnel length in the influence area of the ductile iron pipes (see Figure 11, area shown in orange colour).

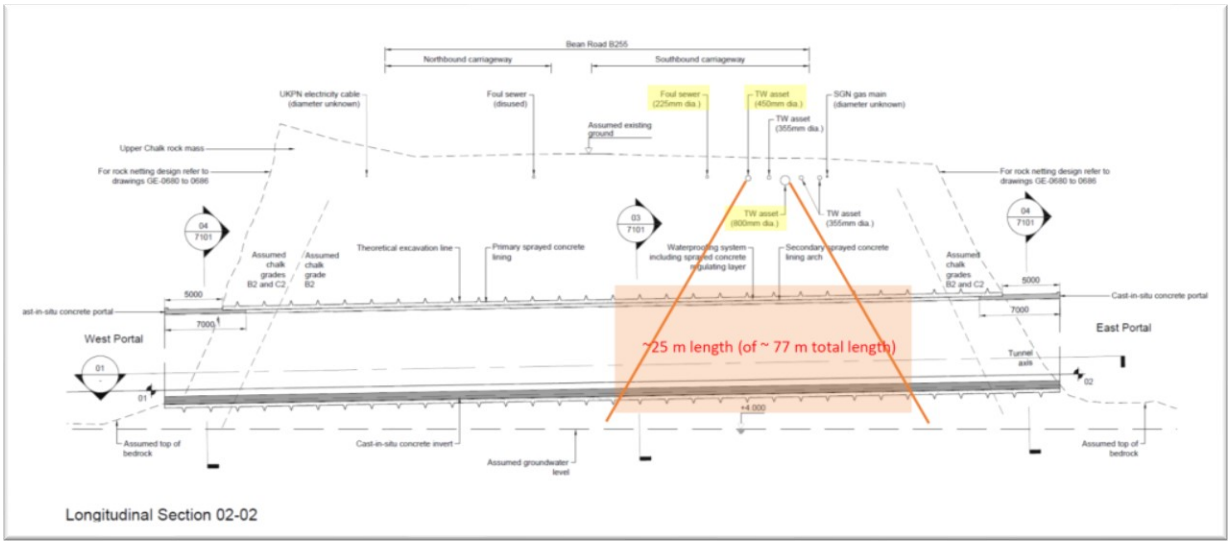


Figure 11 Tunnel length in the influence area of the ductile iron pipes.

It is to be noted that detailed impact assessments have not been carried out for the UK Power Networks and SGN assets. However, the ground movements and axial strains these assets are subjected to are the same as presented for other utilities in this report.

6. References

6.1. Meetings

[1] Meeting between Thames Water, Kent County Council, Balfour Beatty, TGP and DSP, 13th October 2020.

[2] Meeting between Thames Water, Kent County Council, Balfour Beatty, TGP and DSP, 19th November 2020.

[3] Meeting between Thames Water, Kent County Council, Balfour Beatty, TGP and DSP, 19th February 2021.

[4] Meeting between Thames Water, Kent County Council, Balfour Beatty, TGP and DSP, 23rd March 2021.

[5] Meeting between Thames Water, Kent County Council, Balfour Beatty, TGP and DSP, 06th May 2021.

6.2. Project documents

[6] Thames Water, Guidance for working near our assets

[7] Thames Water searches: JAC19039_DURS

[8] Thames Water searches: DA9 9SX clean GIS

[9] Thames Water searches: DA9 9SX waste GIS

[10] Jacobs drawing no. B2350500-JAC-DD-TUN-DR-C-Zz_70_20-1009

[11] Jacobs drawing no. B2350500-JAC-DRG-HA-0017

[12] Technical Note – Tunnel Design Assessment

[13] Technical Note – Rock Stability Assessment Draft

[14] Dr. Sauer and Partners drawing no. BRT-TG-XX-TU-DR-TU-7101

6.3. Literature

[15] P.C. Watson, C. Warren, J.C. Hurt and C. Eddie (1999). The design of the North Downs Tunnel. Tunnel Construction & Piling 99. Institution of Mining & Metallurgy.

[16] G. Patullo (1995). Construction of the A20 Round Hill tunnels, Kent. Civil Engineering 108, 54-63.

[17] A. Liew, ICE Publishing (2016). Observations of tunnelling effects on a large diameter jointed concrete pipe on Crossrail contract C310 at North Woolwich, Newham, London. Crossrail Learning Legacy website.

[18] Bracegirdle A., Mair R. J., Nyren R. J. and Taylor R. N. (1996). A methodology for evaluating potential damage to cast iron pipes induced by tunnelling. Geotechnical aspects of underground construction in soft ground. pp 659-664. Balkema: Amsterdam.

[19] O'Reilly M. and New B. (1982). Settlements above tunnels in the United Kingdom - their magnitude and prediction. Proc. Tunnelling 82, pp 173-181. Instn. Min. and Metal. London. (Reprinted in May 2015 Tunnels and Tunnelling).

[20] I. Bjorklund (2001). Structural design of pressure pipes. Water Science and Technology: Water Supply Vol 1 No 3 pp 107–115.

[21] R. J. Mair, R. N. Taylor and A. Bracegirdle (1993). Subsurface settlement profiles above tunnels in clays. Geotechnique 43, No. 2, pp 315-320.

[22] B. New (2019). Tunnel construction impacts on utility pipelines. Tunnels and Tunnelling February 2019 pp 20-42.

[23] F. Mirada, R. Impiumi and Z. Farook. Method for assessing damage induced in utilities due to ground movements from major infrastructure projects.

Appendix

Appendix A – Additional contour plots

Appendix B – Verification of the assumed cohesion value

Appendix C – Calculation of the limit of settlement

Appendix D – Drawing Instrumentation and Monitoring Bean Road

Appendix A - Additional Contour Plots

This appendix provides additional contour plots, including:

- Contour plots for stages 2 and 4 requested by Thames Water [5];
  (note: stages 3 and 5 are available in the main text)
- Contour plots for stages 3 and 5 without SCL lining requested by Thames Water [5]; and
- Contour plots for sensitivity analyses carried out on the cohesion value.

Stages 2 and 4

(Note: contour plots for stages 3 and 5 are available in the main text)

Stage 2:

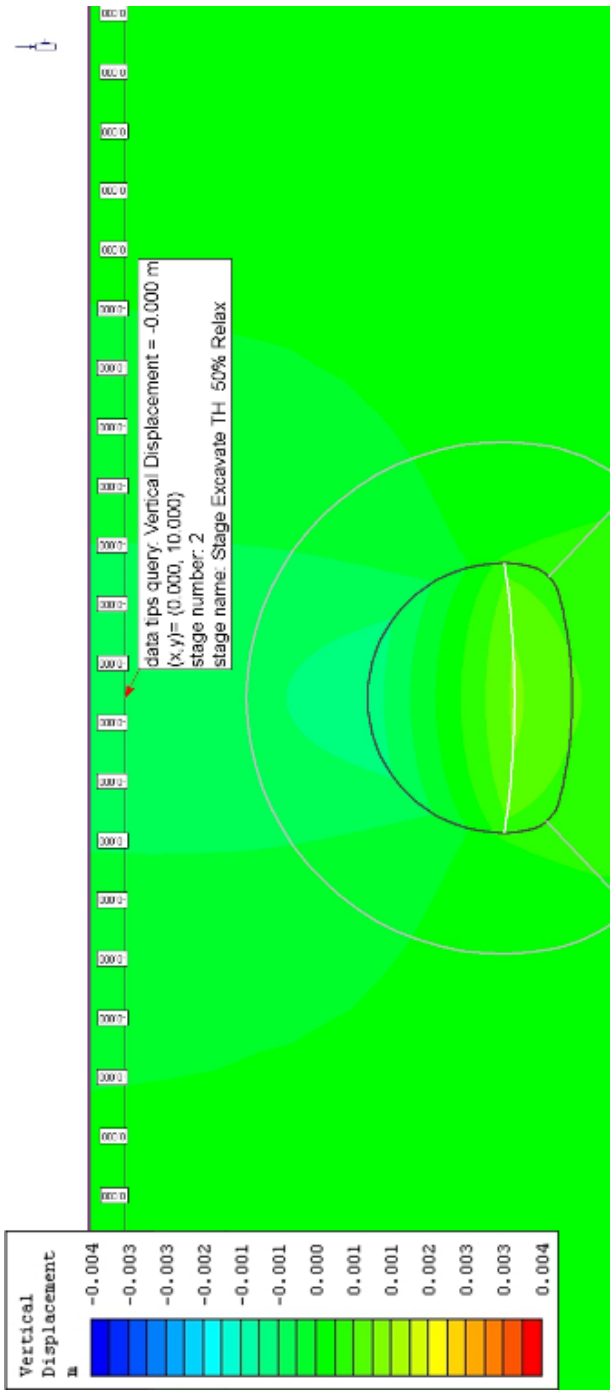


Figure A 1 Contour plot for stage 2.

Stage 4:

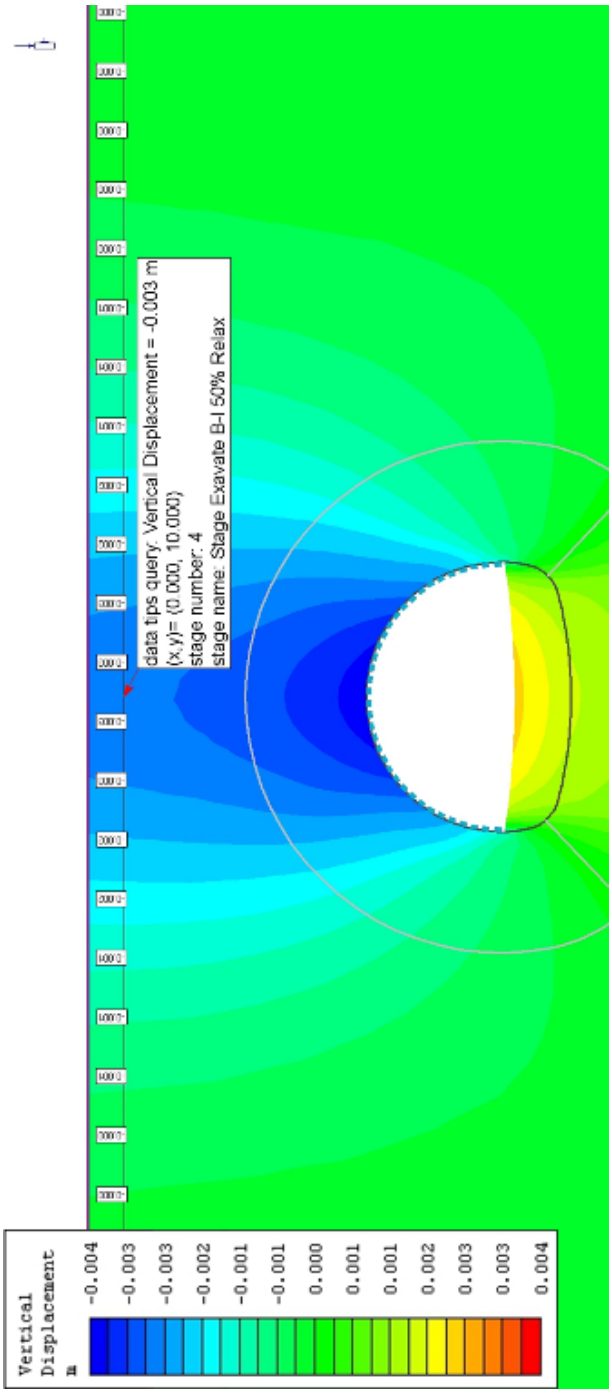


Figure A 2 Contour plot for stage 4.

Stages 3 and 5 without SCL lining

Stage 3 (no lining):

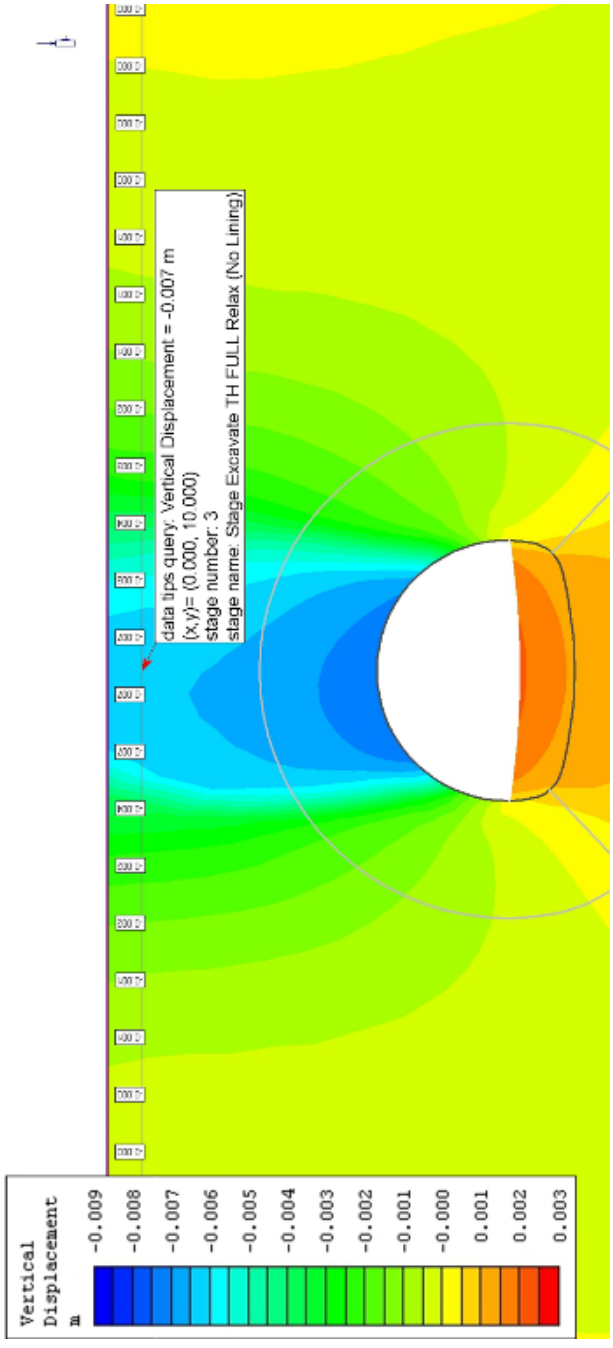


Figure A 3 Contour plot for stage 3 without SCL lining.

Stage 5 (no lining):

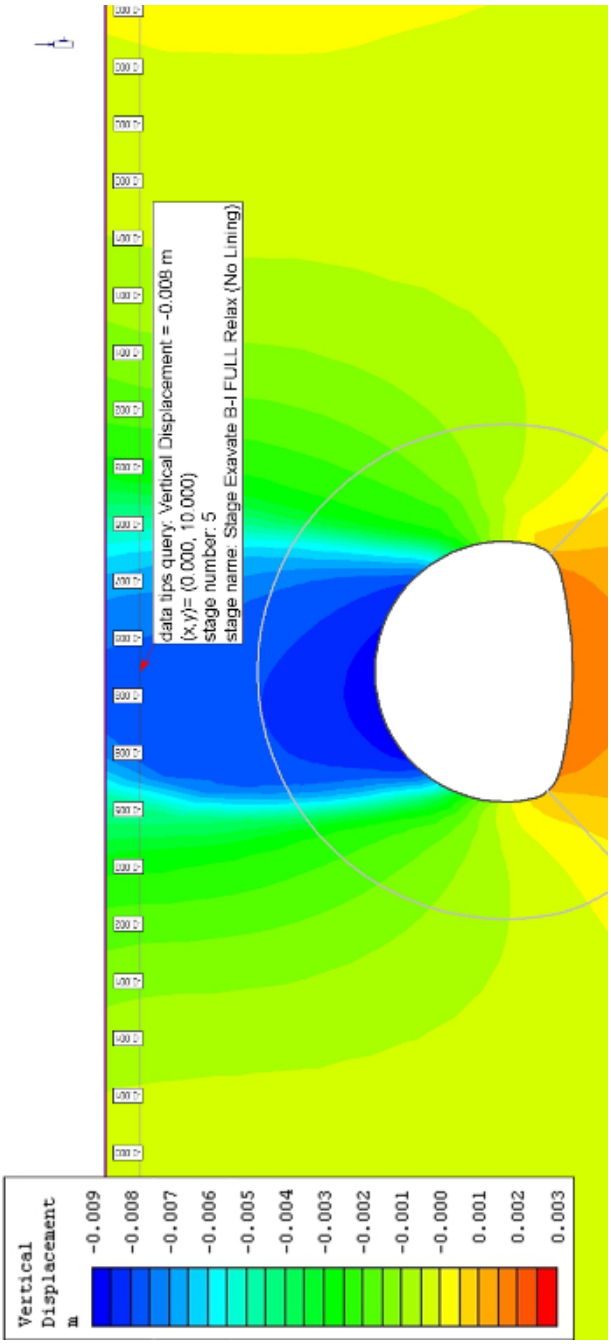


Figure A 4 Contour plot for stage 5 without SCL lining.

Sensitivity analyses

Stage 5 (cohesion of 40kPa):

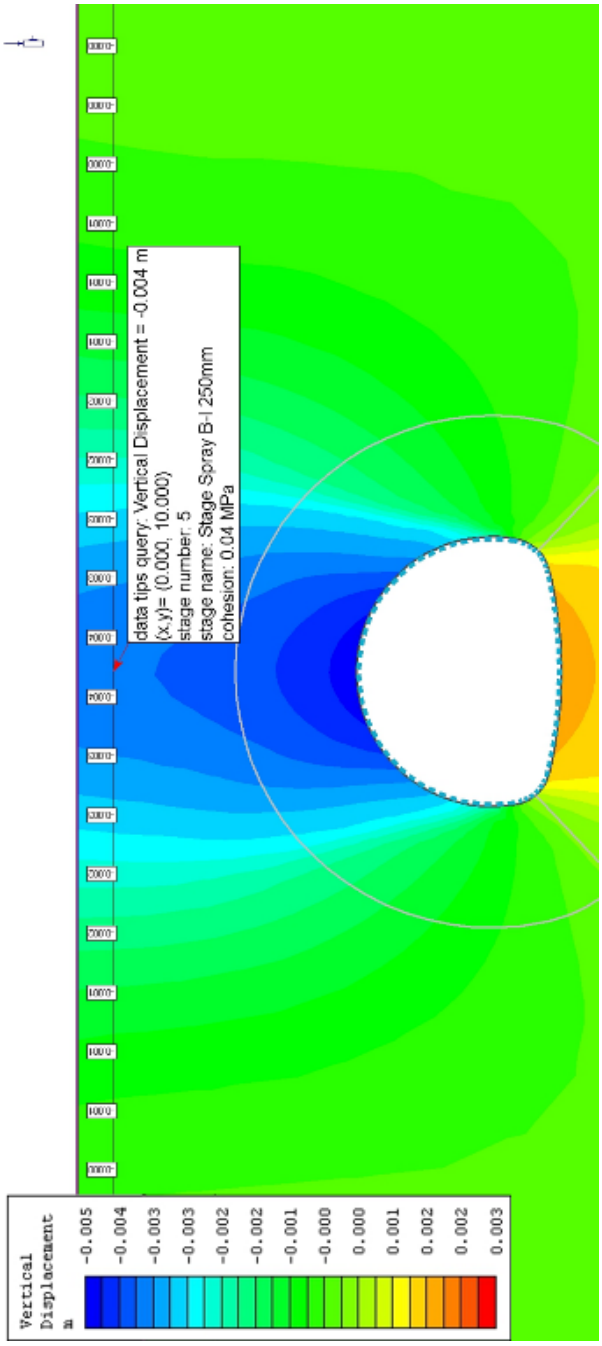


Figure A 5 Contour plot for stage 5 with cohesion of 40kPa.

Stage 5 (cohesion of 30kPa):

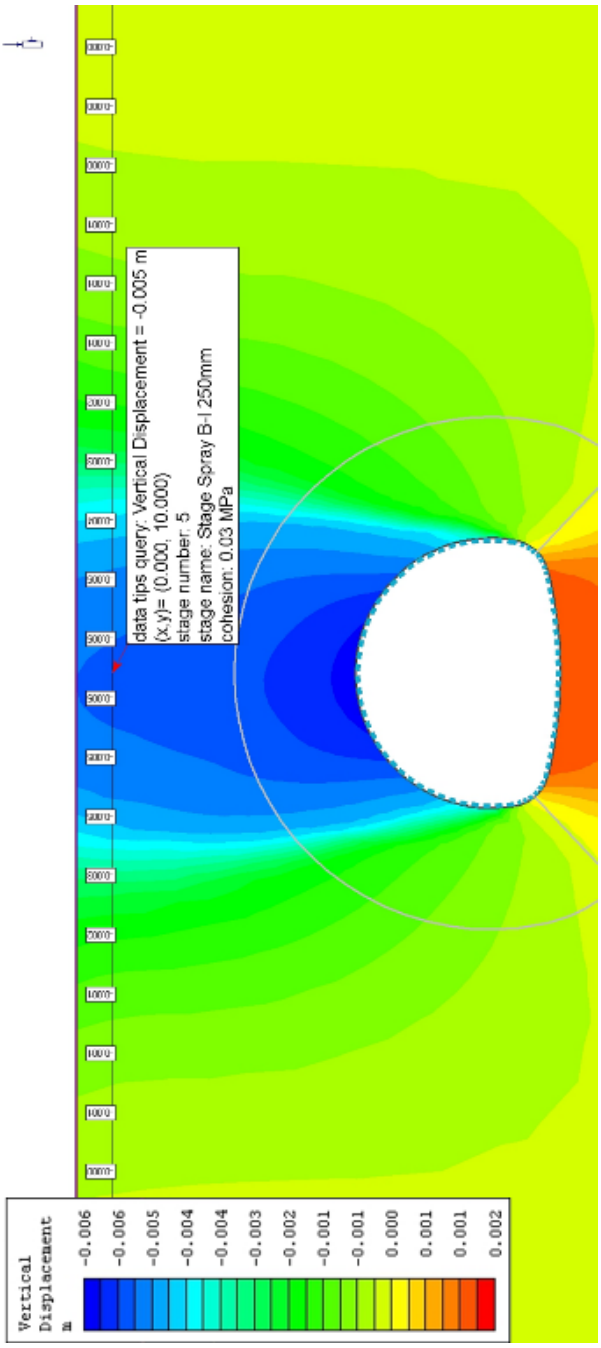


Figure A 6 Contour plot for stage 5 with cohesion of 30kPa.

Stage 5 (cohesion of 20kPa):

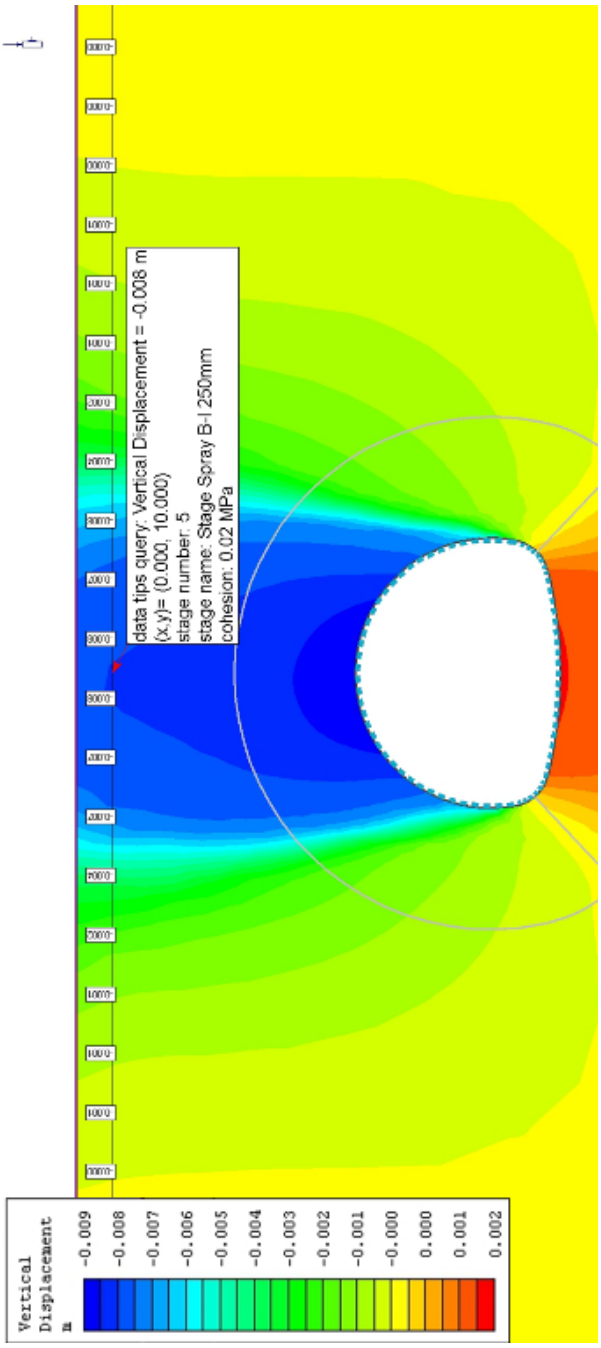


Figure A 7 Contour plot for stage 5 with cohesion of 20kPa.

Stage 5 (cohesion of 15kPa):

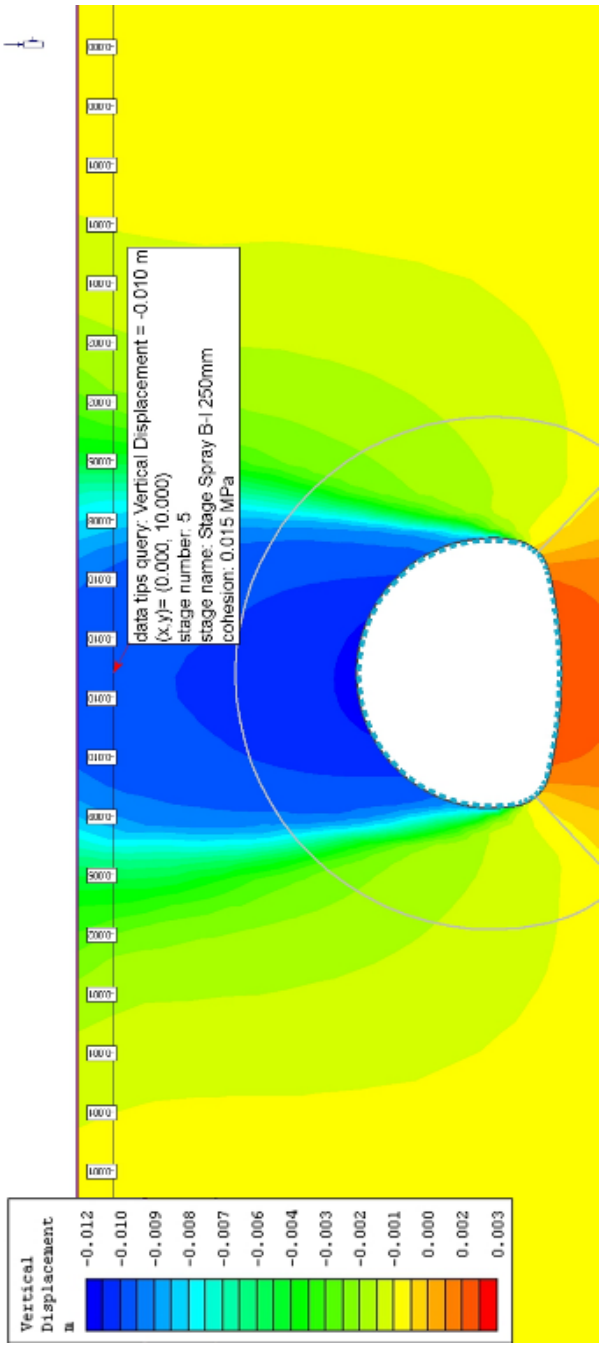


Figure A 8 Contour plot for stage 5 with cohesion of 15kPa.

Stage 5 (cohesion of 10kPa):

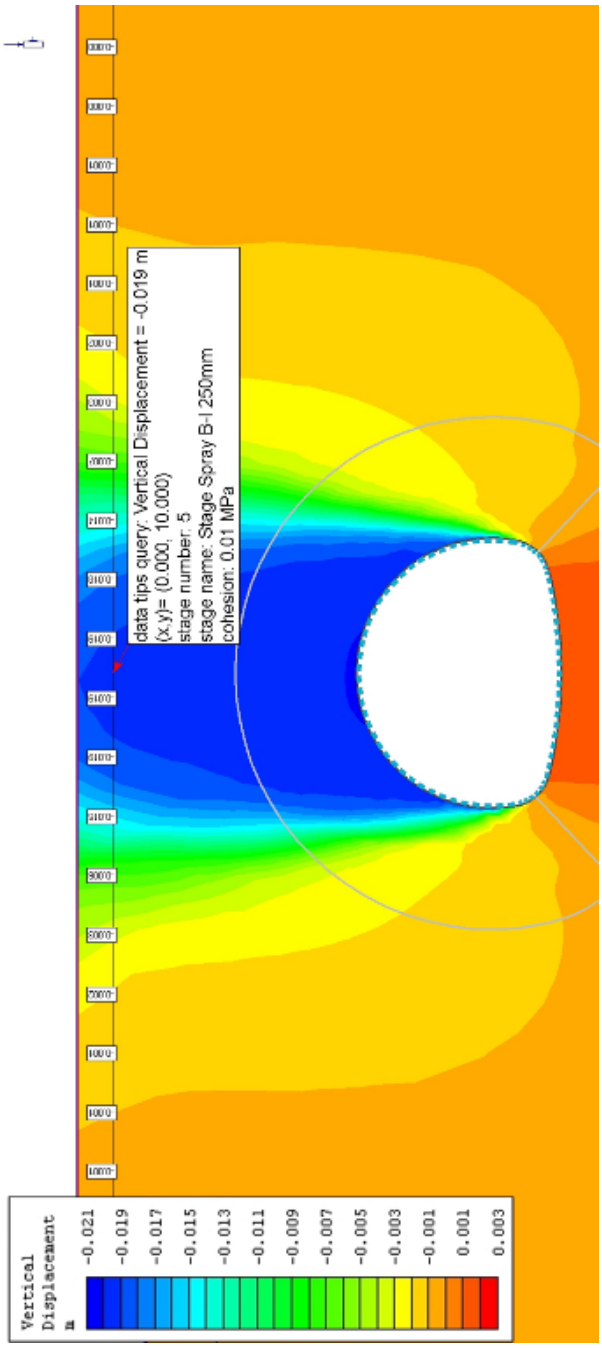


Figure A 9 Contour plot for stage 5 with cohesion of 10kPa.

Stage 5 (cohesion of 5kPa; note that the model was unstable with convergence issues):

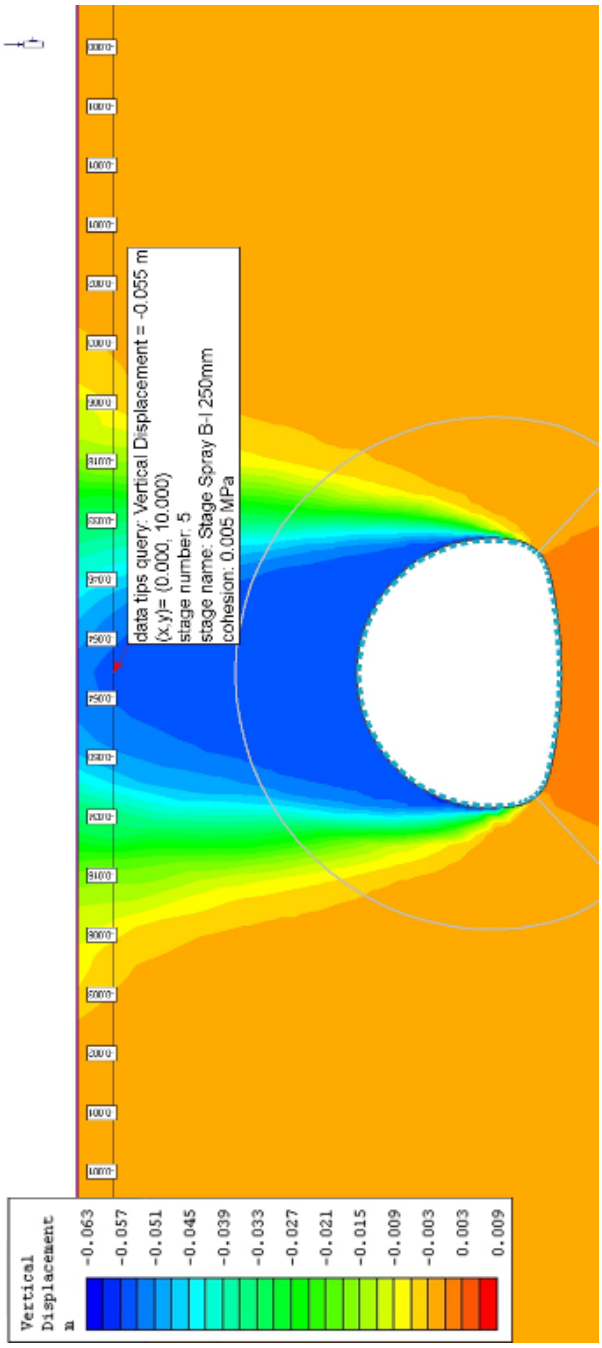


Figure A 10 Contour plot for stage 5 with cohesion of 5kPa.

Stage 5 (cohesion of 5kPa + temporary invert):

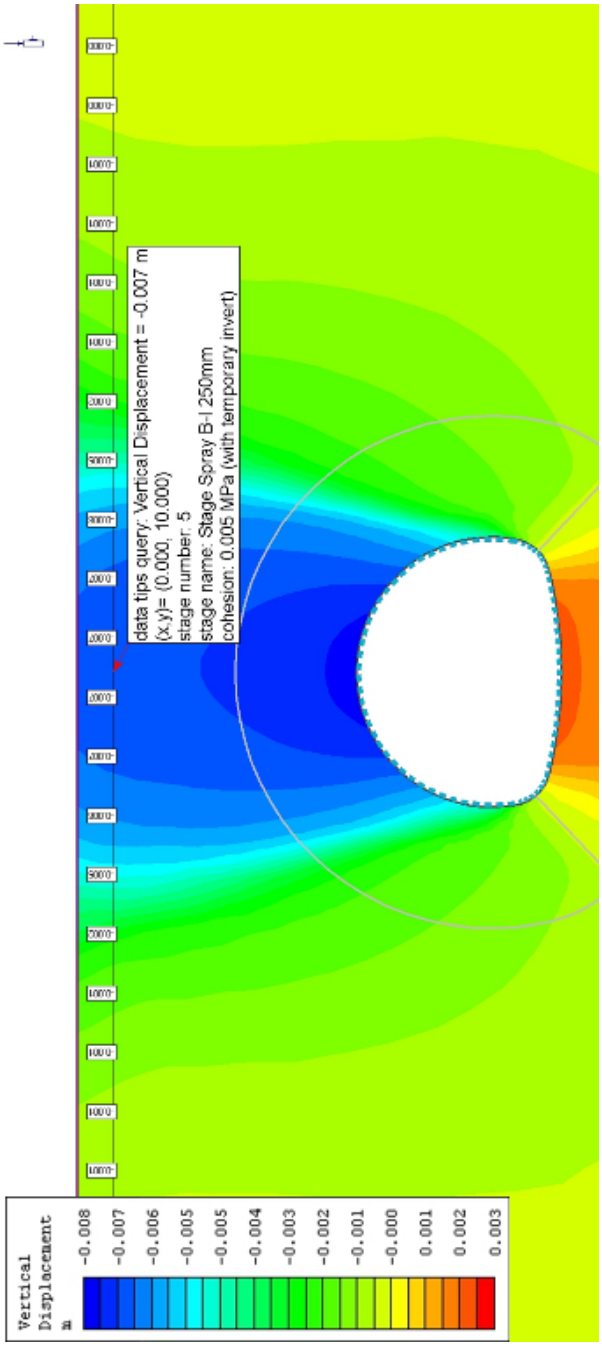


Figure A 11 Contour plot for stage 5 with cohesion of 5kPa if temporary invert is used.

Appendix B – Verification of the Assumed Cohesion Value

The current slopes of the chalk spine were analysed with the aim to assess a suitable cohesion value for the chalk spine. This assessment confirmed the considered value of approximately 50kPa as shown in Figure B 1.

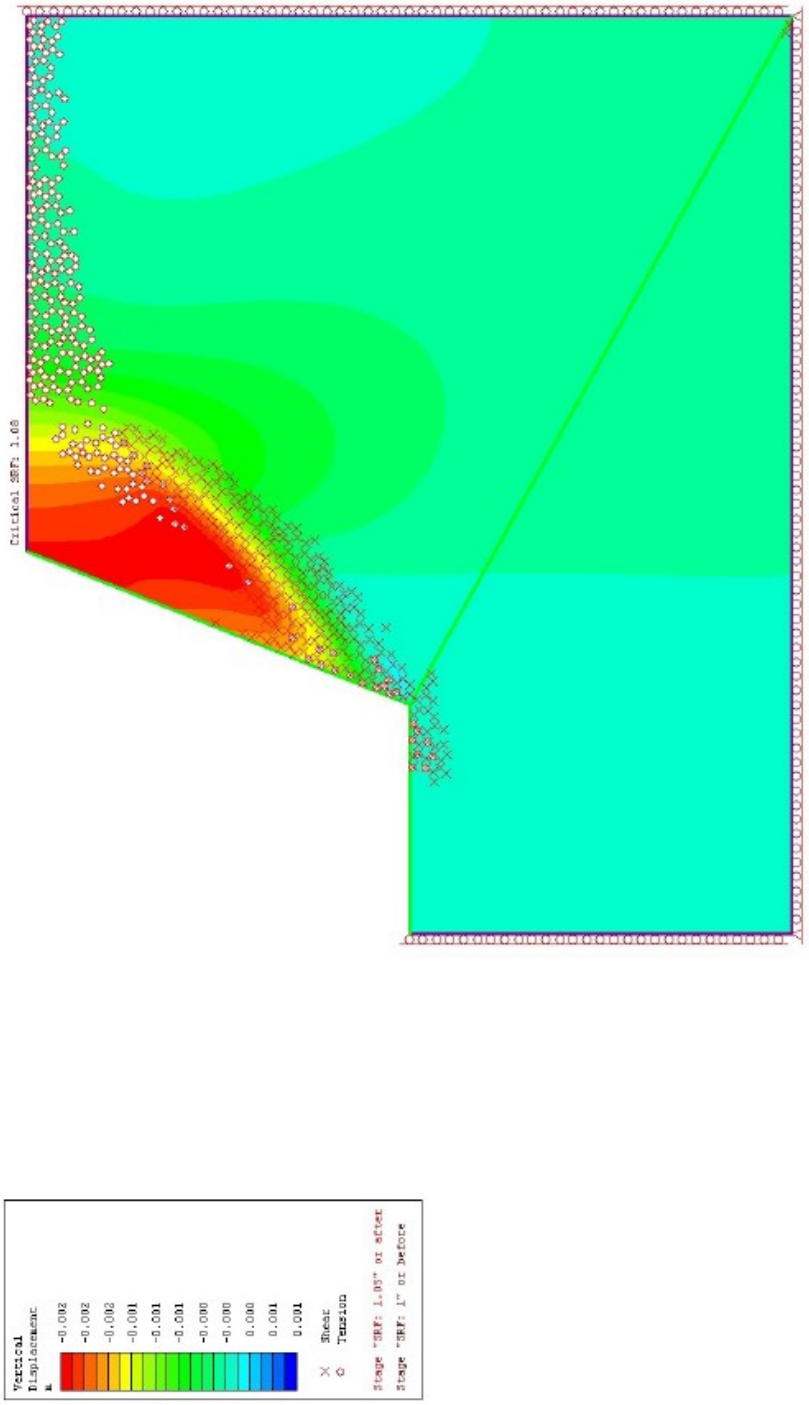


Figure B 1 Analysis of the chalk spine.

Appendix C – Calculation of the Limit of Settlement

Based on section 4 of the main text, the 800mm ductile iron pipe has the lowest ratio of acceptance criteria to predicted value of 4.38. This is based on an assessment criteria of 700με and a predicted value of 160με shown in Table 6 of the main text.

The 700με acceptance criteria is provided by Thames Water. In order to assess the maximum allowable movement which will result in a ratio of acceptance criteria to predicted value of 1.0, the maximum allowable value of increase in strain is assumed as 700με.

As described in section 3 of the main text:

$$\varepsilon_{tot} = \varepsilon_{axial} + \varepsilon_b$$

$$\varepsilon_b = \frac{D}{2R}$$

$$R_n = \delta^2 / \{2 \cdot S_{v,n} - (S_{v,n-1} + S_{v,n+1})\} \; for\ a\ generic\ point\ y = yn$$

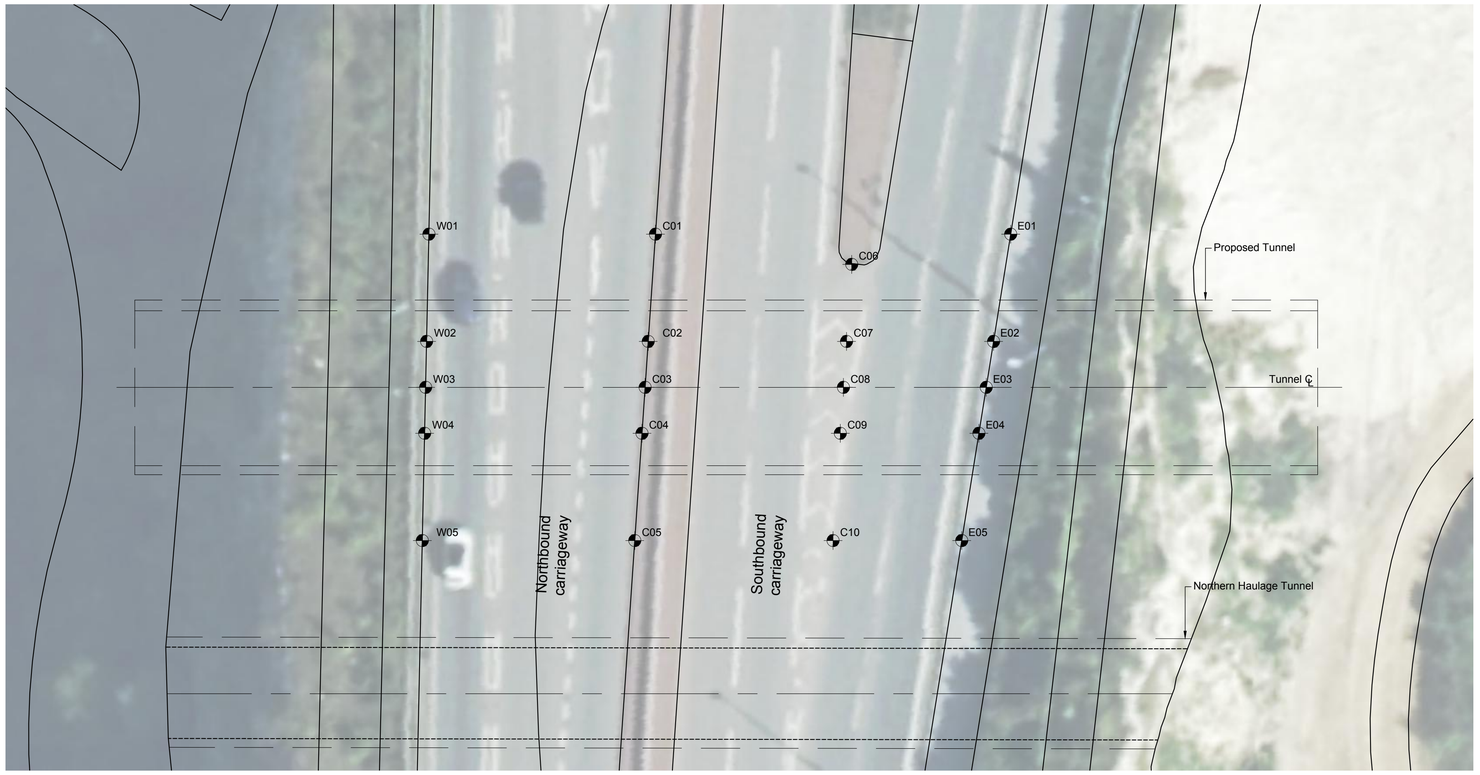
$$\varepsilon_{axial} = RF \cdot \varepsilon_h$$

$$\varepsilon_h = \frac{dS_h}{dy} = \frac{S_v}{Z_0 - Z_p} \cdot \left(1 - \frac{y^2}{i^2}\right)$$

(Note: see main text for the definitions of the symbols)

It can be seen that ε_{tot} is linearly related to S_v. Since the ratio of predicted values and acceptance criteria is 4.38, a maximum allowable movement of 12mm is proposed (4.0 times predicted value, conservative).

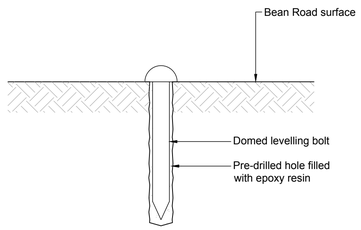
Appendix D – Drawing Instrumentation and Monitoring Bean Road



Plan 01-01
Bean Road

Scale 1:200

Setting-Out Coordinates		
ID	Easting	Northing
W01	558477.221	173510.939
W02	558476.396	173503.988
W03	558476.042	173501.008
W04	558475.688	173498.029
W05	558474.862	173491.078
C01	558491.946	558491.946
C02	558490.799	173502.590
C03	558490.307	173499.624
C04	558489.816	173496.658
C05	558488.668	173489.736
C06	558504.511	173506.313
C07	558503.687	173501.340
C08	558503.195	173498.373
C09	558502.704	173495.407
C10	558501.556	173488.486
E01	558515.032	173507.272
E02	558513.243	173500.413
E03	558512.476	173497.473
E04	558511.710	173494.533
E05	558509.920	173487.674



Detail 01
Levelling Bolt

Scale 1:1

20mm 0 20mm 40mm 60mm 80mm 100mm
SCALE 1:1

4m 0 4m 8m 12m 16m 20m
SCALE 1:200

DO NOT SCALE FROM THIS DRAWING

NOTES

1. FOR TUNNELLING SPECIFIC NOTES REFER TO DRAWING BRT-TG-XX-TU-DR-TU-7001.
2. ALL PRODUCTS / INSTRUMENTS SHALL BE USED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.
3. LEVELLING BOLTS (OR SIMILAR APPROVED) SHALL BE INSTALLED TO MONITOR VERTICAL MOVEMENTS OF THE BEAN ROAD.
4. LEVELLING BOLTS SHALL BE CORROSION RESISTANT DOMED METAL BOLTS WITH RIBBED SHANK FIRMLY FIXED WITH EPOXY RESIN IN CLEANED DRILLED HOLES.
5. LEVELLING BOLTS SHALL BE INSTALLED AT THE INDICATED LOCATIONS CONSIDERING AN INSTALLATION TOLERANCE OF 1 METER IN ANY DIRECTION.
6. LEVELLING BOLTS SHALL NOT BE FIXED IN LOCATIONS OR IN SUCH A MANNER THAT COULD SUBJECT THEM TO WEAR OR DISTURBANCE FROM PEDESTRIAN OR VEHICULAR TRAFFIC.
7. BASELINE READINGS OF ALL LEVELLING BOLTS SHALL BE TAKEN BEFORE TUNNEL CONSTRUCTION START.
8. THE REGULAR READING FREQUENCY DURING TUNNEL CONSTRUCTION SHALL BE 1 SET OF READINGS EVERY 24 HOURS.
9. ANY INCREASE OR DECREASE OF READING FREQUENCIES SHALL BE AGREED AT THE DAILY REVIEW MEETING.
10. MONITORING DATA SHALL BE REVIEWED AGAINST PREDICTED BEAN ROAD MOVEMENTS THROUGHOUT CONSTRUCTION TO VERIFY PREDICTED MOVEMENTS.
11. BACKGROUND MAPPING FROM BING MAPS AND IS APPROXIMATE.
12. THIS DRAWING CONTAINS THE FOLLOWING REFERENCES:
A120036_A1_Drawing_Border
OS_TQ5873
BRT-TG-XX-ZZ-M2-TU-7106
BRT-TG-XX-ZZ-M2-TU-7400

REV.	BY	CHK'D	APP'D	DATE	DESCRIPTION
P02	LCH	AFE	---	11/03/21	FOR STAGE APPROVAL
P01	LCH	AFE	---	19/02/21	FIRST ISSUE

CODE	DRAWING STATUS
S4	FOR STAGE APPROVAL

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BEAN TUNNEL
TUNNEL AND HIGHWAY DETAILED DESIGN

TUNNELS INSTRUMENTATION AND MONITORING BEAN ROAD

SHEET 1 OF 1

RESIDUAL HAZARD	SUGGESTED CONTROL MEASURE

SIGNIFICANT RESIDUAL HAZARDS

NOTE: The above hazards do not include every hazard or assumption, but identify significant residual construction hazards that are not likely to be obvious to a competent contractor and those that could be difficult to manage effectively. Refer also to the designer's risk documentation.

Dr. SAUER & PARTNERS

DRAWING. No.
BRT-TG-XX-TU-DR-TU-7401

DRAWN : LCH	DESIGNED : AFE	REVISION
SCALE : AS SHOWN	ORIGINAL SIZE : A1	P02